Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte
Band: 56 (1987)

Artikel: Instrumentation of glued segmental box girder bridges
Autor: Barr, Ben I.G. / Waldron, Peter / Evans, H. Roy
DOI: https://doi.org/10.5169/seals-43563

Instrumentation of glued segmental box girder bridges

Appareillage de mesure installé sur des ponts à caissons segmentés collées

Messgeräteeinrichtung auf geleimten Segmentkastenträgerbrücken

Ben I.G. BARR
Senior Lecturer
in Civil and
Structural Engineering
University College,
Cardiff, UK
Ben Barr is a chartered civil engineer engaged in concrete materials research. His main interests are in the fracture processes in concrete and the fracture characteristics of fibre reinforced concrete materials.

Peter WALDRON
Lecturer in Civil
Engineering,
University of Bristol,
Bristol, UK
Peter Waldron is a chartered civil engineer engaged in structural concrete research. His main interests are in instrumentation, interpretation of experimental results, prototype testing and analysis of thin-walled structures.

H. Roy EVANS
Professor of Civil
and Structural
Engineering,
University College,
Cardiff, UK
Roy Evans is head of department of civil and structural engineering at Cardiff. He is interested in composite construction and has conducted extensive research into all types of bridge and thin-walled structures.

SUMMARY
Glued segmental construction of post-tensioned concrete box girder bridges is a relatively new technique in the UK. Over the last year, three major bridge structures of this type have been instrumented to yield much needed information on (i) early shrinkage strains; (ii) time dependent effects, such as creep and loss of prestress; (iii) elastic strains during erection and early service life; (iv) temperature; and (v) formwork pressures during casting. The instrumentation schemes are briefly described and some early results are presented.

RESUME
La construction des ponts à caissons segmentés collés en béton postcontraint est une technique relativement récente au Royaume Uni. L'an dernier les structures de trois des plus importants ponts de ce type ont été équipées d'instruments de mesure afin de fournir des informations de première nécessité sur: (i) les dilatations de retrait primaire; (ii) les effets temporels, tels que le fluage et les pertes de précontrainte; (iii) les contraintes élastiques durant le montage et la première mise en service; (iv) les températures; (v) les pressions de coffrage durant le bétonnage. On décrit brièvement les schémas d'installation des instruments et on présente quelques premiers résultats.

ZUSAMMENFASSUNG
Im Vereinigten Königreich stellt die Herstellung von geilemten Kastenträgerbrücken ist eine ziemlich neuartige Herstellungstechnik dar. Während dem letzten Jahr sind drei solche grössere Brückentragwerke mit Messgeräte ausgestattet worden, um sehr mangelnde Nachweise zu erlanger über: (i) die Anfangsschwinddehungen; (ii) die zeiabhängige Wirkungen, wie das Kriechen und die Vorspannuns- verluste; (iii) die elastischen Spannungen während der Konstruktion und der ersten Dienststellung; (iv) die Temperaturen; (v) die Schalenpressungen während der Betonierung. Dieser Artikel beschreibt kürzlich die Messeinrichtungsschemas und präsentiert einige erste Ergebnisse.

1. INTRODUCTION

Segmental construction of concrete bridges was introduced into Europe in the 1950's. The method is particularly suitable for medium to long span bridges and is now commonly used in all parts of the world. For example, in North America, where the advantages inherent in this form of construction have long been recognised, over one hundred such bridges have been built since the 1970's. In the UK, the first segmental bridge was the Clifton Bridge, Nottingham, completed in 1957. Although over thirty such bridges have been built in the UK since then, the majority of these have been completed in the last decade. The increasing use of segmental techniques in recent years has occurred for a variety of reasons. These include improvements in materials, better analytical techniques and a growing awareness of the overall enonomic benefits.

1.1 Segmental construction techniques

The term segmental construction is very broad and refers to any concrete bridge structure that is cast in a number of distinct longitudinal segments. Several alternative techniques have evolved over the years. These are usually categorised according to the method of casting and erection of the segments [1] and include the (a) balanced cantilever method; (b) progressive placing method; (c) span-by-span method; and (d) incremental launching method.

1.2 Project description

By far the most significant construction technique is that employing balanced cantilevers in which the cost penalties of using segmental construction are more than offset by the savings brought about by the reduction in falsework. Fig. 1 provides an indication of the frequency with which each technique has been employed in North America, and the span lengths for which each is best suited [2]. It is apparent that 85% of all segmental bridges built to date are of balanced cantilever construction. Precast and cast in situ segments have been employed to about the same degree. In the UK, the pattern is similar with approximately two thirds of segmental bridges being of the balanced cantilever type of construction.

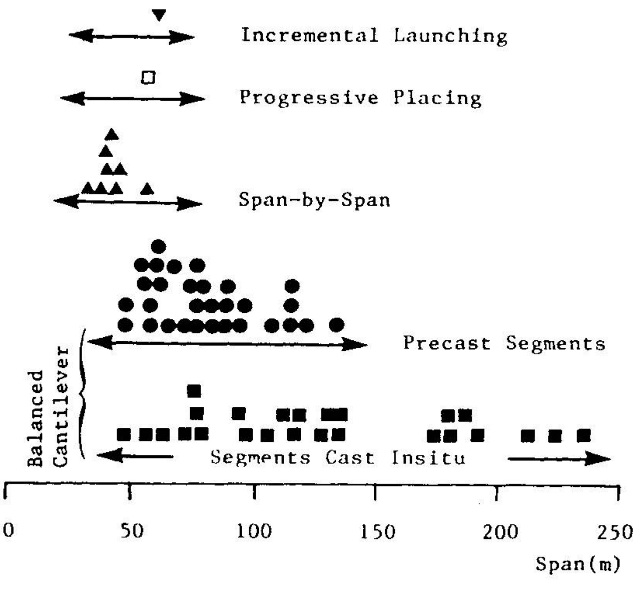


Fig. 1 Segmental bridge types

1.2.1 Limitations

The research project described here concentrates on the balanced cantilever method as the predominant structural technique. More specifically the investigation is centred on glued segmental construction in which match-cast precast segments are prestressed together separated by a thin layer of epoxy resin. This technique is becoming increasingly competitive especially for medium span viaducts. Despite these restrictions much of the research should be of equal value to the other forms of segmental construction.

1.2.2 Objectives

One of the most important considerations in the design of segmental bridges by balanced cantilever is the effect of the time-dependent properties of the materials employed in construction. These include shrinkage and creep of the concrete, and relaxation in the prestressing steel. Since each segment is cast from a different batch of concrete on a different day, each segment may then be expected to respond uniquely to the application of construction loads during and immediately after erection. Moreover, since each balanced cantilever may take several weeks to erect, continuity at midspan will not generally be achieved until one half-span has been subjected to much higher levels of creep, shrinkage, etc..

In bridges employing cast insitu segments or precast segments with thick mortar joints there is some opportunity for adjusting the vertical profile during erection. However, in glued segmental bridges formed from match-cast segments, vertical profile must be decided at the time of casting and is very difficult to adjust during erection. For this reason all deformations due to time-dependent effects and elastic response under varying construction loads must be accurately calculated at the design stage.

A further consideration in all thin-walled concrete bridges is the effect of differential temperature distributions across the wall thickness. This can be of particular significance in box girders for which the volume of air enclosed inside the box may have the effect of accentuating the thermal gradients across the walls. In such cases, deformation may not be restricted to longitudinal movement only but may include significant longitudinal and transverse bending effects.

Finally, in segmental bridges of this type employing precast segments, dimensional accuracy during casting is of paramount importance. Deformation of formwork due to self-weight of wet concrete and the vibrations created during compaction must therefore be closely controlled.

In summary the main objectives of the research programme are to investigate the following:

(i) early shrinkage strains in concrete segments;
(ii) time-dependent effects, such as creep and loss of prestress;
(iii) elastic strains and deflections during erection and early service life;
(iv) thermal effects due to temperature differentials across the walls; and
(v) formwork pressures caused by external vibration during casting.

A dearth of information exists in these areas, particularly with respect to the climatic and environmental conditions prevailing in the UK. A major initiative has recently been established to investigate these effects. Analytical and computational aspects of the research are being verified and calibrated by monitoring the performance of three major glued segmental bridges. It is the instrumentation and early measurements from this field investigation which are described in some detail here.

2. DESCRIPTION OF INSTRUMENTED BRIDGES

Three glued segmental bridges were identified for instrumentation, each with a very different structural configuration. The first bridge was straight in plan with varying section depth; the second and third had almost constant cross-sectional depth but displayed very different levels of horizontal curvature.

2.1 River Torridge Bridge

The River Torridge Bridge, located 1km north of Bideford, North Devon, forms part of the 8.4km Bideford bypass on the A39 trunk road between Taunton and Freddon. Completed in May 1987 it carries two lanes of traffic 29m above mean high water level over the Torridge tidal estuary.

The bridge consists of eight continuous spans, each up to 90m in length, with a total length of 645m. The general arrangement and elevation of the bridge is shown in Fig. 2. The superstructure is straight in plan and is formed from 251 segments weighing up to 105 tonnes each. Its cross-section, Fig. 3(a), is that of a single cell box girder with wide side cantilevers, varying in depth from 6.1m at the supports to 3.1m at midspan.

Segments were match cast on site by the short-line method at a rate of approximately five per week. After curing for several months, each unit was transported to the western end of the partly completed bridge and erected using a purpose built launching girder itself weighing 150 tonnes. As each segment was positioned, a thin layer of epoxy resin was applied by hand to the matching faces immediately prior to prestressing, thus ensuring water-tightness and a uniform transfer of stress across the joint.

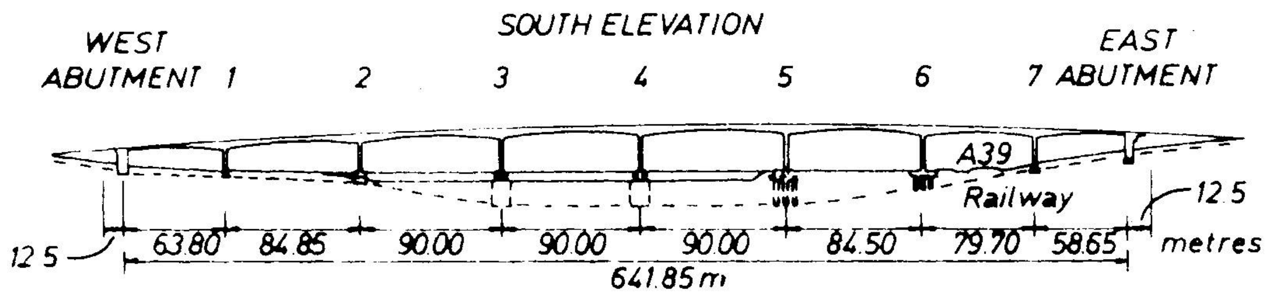


Fig. 2 Elevation of River Torridge Bridge.

2.2 Grangetown Viaduct

The second and third viaducts to be monitored are both located on the peripheral distributor road currently being constructed around the city of Cardiff. The required alignment, together with the frequency and spacing of the obstacles, made the choice of viaducts a logical solution for this part of the scheme. Both viaducts were designed to carry the Department of Transport's HA loading and were checked for 45 units of HB loading. This design loading was used in order that the viaducts could be used to meet the needs of the adjacent docks and also to cater for future industrial development along the peripheral distributor road.

The Grangetown Viaduct, which is over 1km long, is the longest post-tensioned glued segmental viaduct in the United Kingdom. The twin trapezoidal box girder superstructure is made up of a total of 641 segments weighing between 43.5 and 74 tonnes. Typical cross-sections for a mid-span segment and a pier segment are shown in Fig. 3(b). Between the abutments, each deck is supported by 14 circular 3m diameter columns varying in height from 8.5m to 18m. The columns in turn are supported by means of hexagonal pile caps.

2.3 Cogan Viaduct

The second post-tensioned glued segmental viaduct to be instrumented at this site is the Cogan Viaduct. Although the Cogan Viaduct is shorter in length it

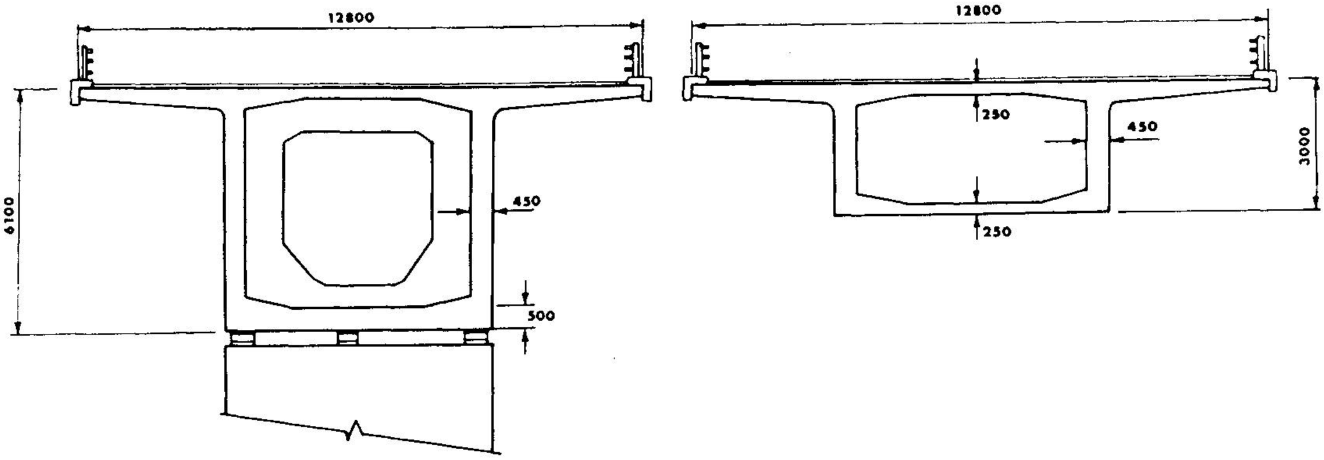


Fig. 3(a) Typical sections through River Torridge Bridge

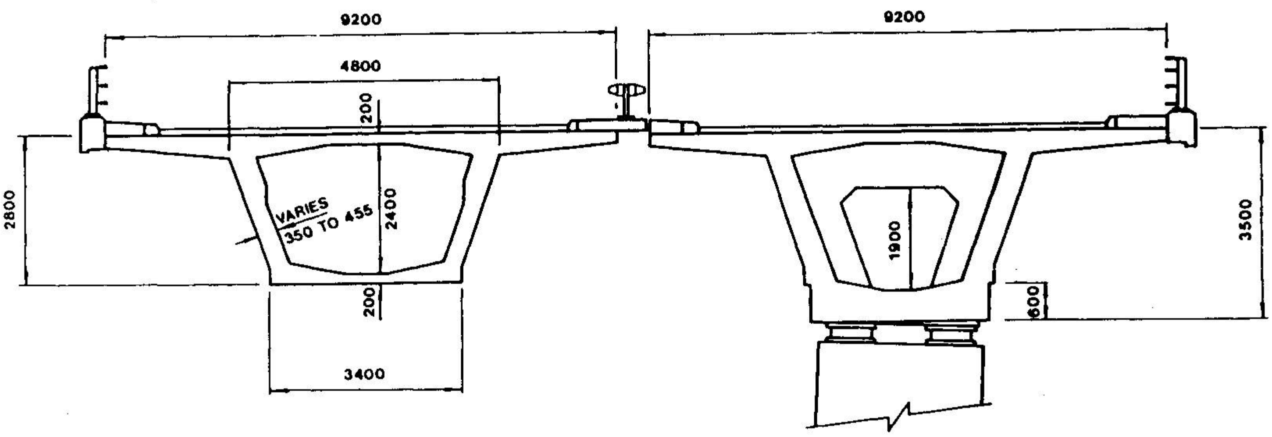


Fig. 3(b) Typical sections through Grangetown Viaduct

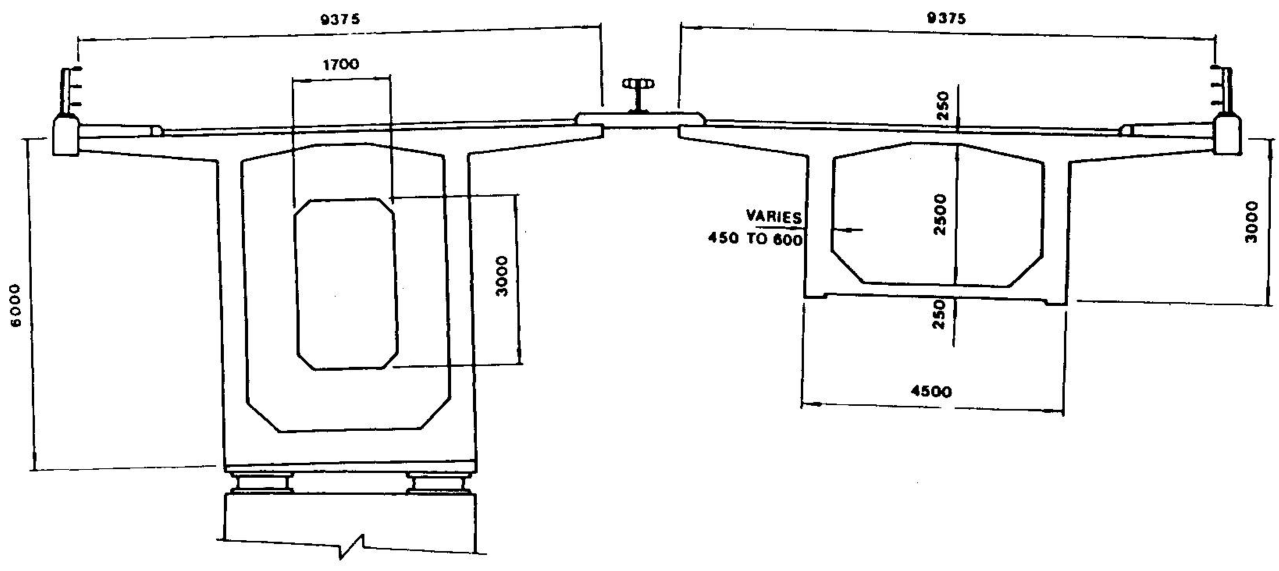


Fig. 3(c) Typical sections through Cogan Viaduct is more complicated than the adjacent structure since it has a main span of 95m and a radius of curvature of only 285m for some of the approach spans. Cogan Viaduct differs from Grangetown Viaduct in that the deck is based on a rectangular box section due to the curved soffit of the 95m main span. Typical cross-sections for a mid-span segment and a pier segment for the Cogan Viaduct are shown in Fig. 3(c). The viaduct has 6 spans for each carriageway and will be built from 300 segments, approximately 2.5m long and weighing between 43 and 117 tonnes. The pier segments to the main span are 6m deep and during their construction some additional monitoring of formwork pressures was carried out.

As for the two previous bridges, the precast deck segments were manufactured on site by the short-line match-cast method. In order to meet the construction programme and ensure independence from weather conditions, a purpose built three-bay portal frame factory was erected. The casting cells in each bay were capable of producing one segment per day and hence only a limited amount of time was available for the installation of the gauges in any given segment. The instrumented spans of the Grangetown and Cogan Viaducts were both erected by crane.

3. INSTRUMENTATION SCHEMES

3.1 Concrete strain

Instrumentation for the measurement of concrete strain was similar in all three bridges. In the River Torridge Bridge four bridge segments were chosen within one of the central 90m spans (Span 5). Their locations, identified in Fig. 4, were designated P4-1E, P4-6E, P4-10E and P4-16E, corresponding to their positions relative to Pier 4. Three further segments within a single half-span were chosen in both the Grangetown and Cogan Viaducts, at approximately mid-span, quarter-span and support sections.

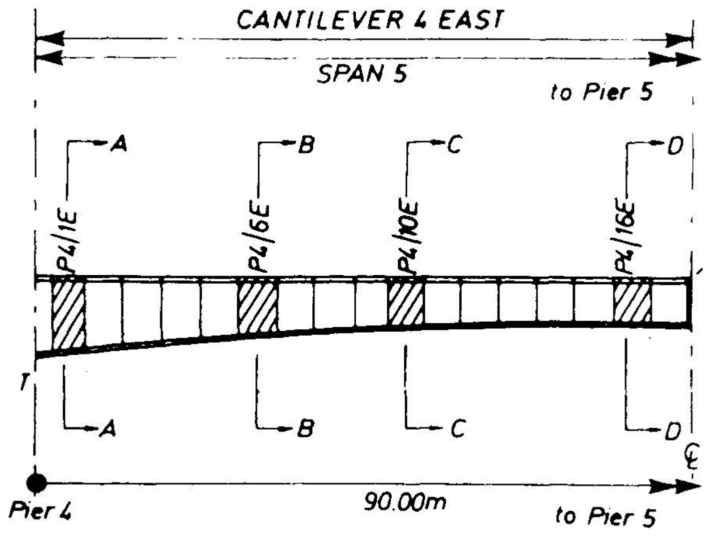


Fig. 4 Instrumented segments

Concrete strain was measured using embedment type vibrating wire strain gauges with a 140mm gauge length. This type of gauge was chosen because of its robustness during installation and casting but also because, unlike electrical resistance gauges, its readings remain unaffected by disconnection. This is essential for field investigations when a complete strain history is required but where it is impracticable to leave data recording equipment permanently attached. This type of gauge has been used extensively for bridge tests and has proven to be reliable and accurate. It has a sensitivity of less than one microstrain and displays very low long term drift.

Each segment was instrumented for measurement of strain at a number of discrete points around the cross-section on the median line of the walls. For segment P4/1E, both single and rosette gauges were used at the positions indicated in Fig. 5. All gauges were fixed to secondary steel attached to the main reinforcement before casting. Axial strains were monitored at six positions, namely the four corners and at the cantilever tips. Three-element rosette gauges were deployed at each of the three intermediate positions in each flange and web element. These yielded additional information on the distribution of axial strain as well as in-plane shear strain around the box section. A similar arrangement was used for segment P4/6E. The remaining two segments in the River Torridge Bridge were instrumented less fully with only single element axial strain gauges.

A similar instrumentation scheme was adopted for the other two bridges. In the three instrumented segments in Grangetown Viaduct, gauges were positioned at eighteen similar locations to those identified previously in Fig. 5. However, in this instance, only the central gauge in each flange and web element was a rosette. In the Cogan Viaduct segments, the six axial gauges in the box corners and cantilever tips were retained with two 120° rosette gauges at intermediate points along each wall element forming the box.

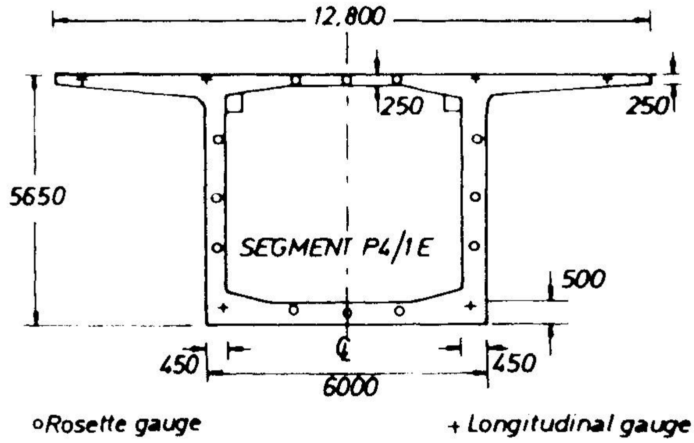


Fig. 5 Strain gauge positions within Segment P4/1E

3.1.1 Instrumentation success rate

In each of the three structures several additional gauges were installed across the thickness of the various wall elements. Since direct strain in this direction is likely to be negligible, these gauges should yield valuable information on long-term concrete shrinkage strains.

A total of 289 vibrating wire strain gauges were deployed in this way in the three structures, as indicated in Table 1. The overall failure rate of approximately 1% was extremely low considering the difficult installation and casting operations.

Structure	Axial	Rosette	Shrinkage	Total	Failures
River Torridge Bridge	40	24	4	116	1
Grangetown Viaduct	42	12	3	81	2
Cogan Viaduct	18	24	2	92	0
Total	100	60	9	289	3

Table 1 Vibrating wire gauge deployment

3.1.2 Material properties

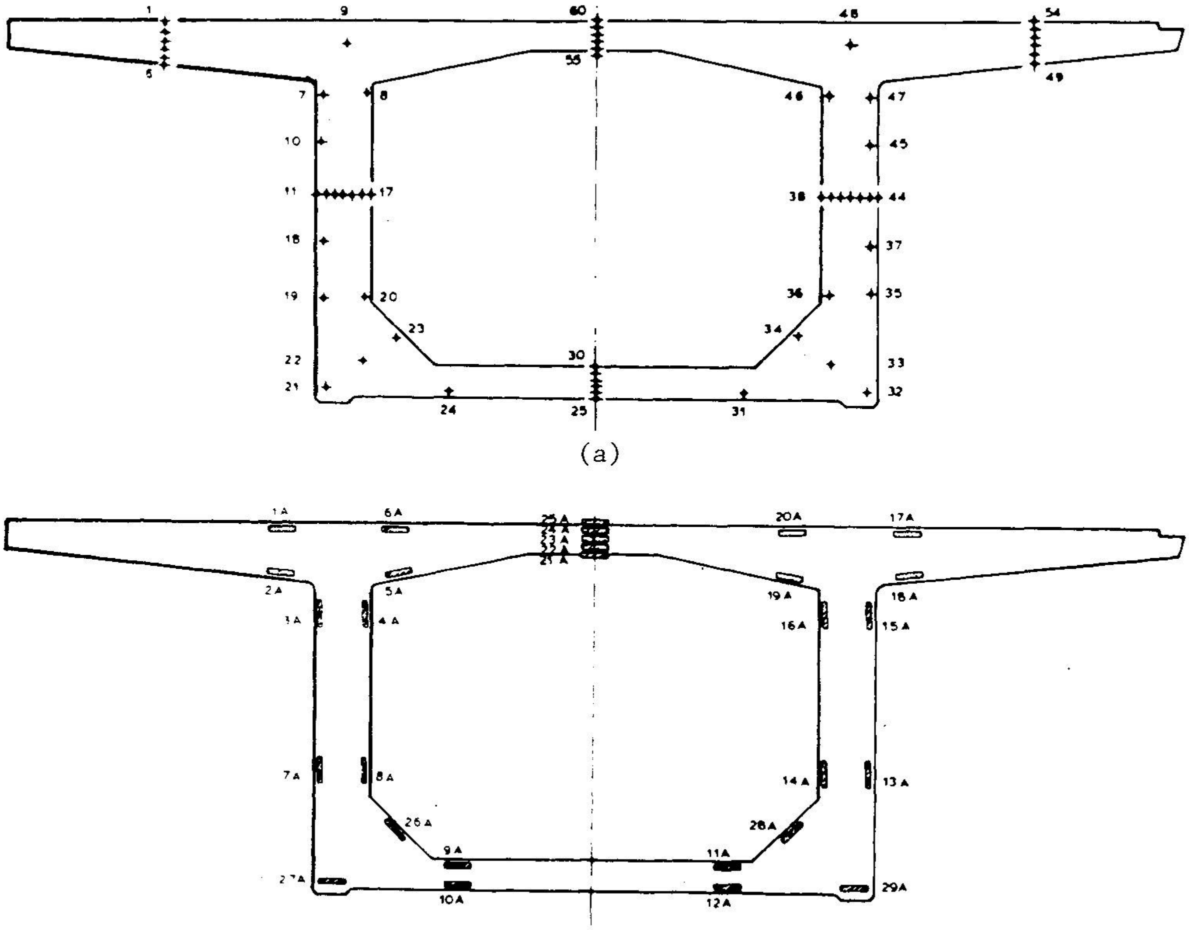
In addition to this programme of monitoring strain behaviour in the bridge structures, a parallel laboratory investigation into the various time-dependent material properties was also initiated. Numerous prism specimens, each with a single axial vibrating wire gauge embedded on the centreline, were manufactured from the batches of concrete used for construction of the instrumented segments. Some have been fully sealed against loss of moisture, for control purposes; the remainder have either been left unsealed or have been partially sealed, to represent more accurately the environmental conditions at the various bridge sites. A number of these specimens are being used to determine

Young's modulus, Poisson's ratio and the coefficient of thermal expansion and their variations with age. The remainder are being used for the long-term assessment of creep and shrinkage effects.

3.2 Temperature monitoring

Copper-constantan thermocouples were used throughout because of their robustness and reliability over long periods of time. They are also relatively cheap and are not greatly affected by the severe environment of the concrete.

The aims of the thermal investigation were twofold. Firstly, to adjust the concrete strain readings obtained at each vibrating wire gauge position for a standard reference temperature; secondly, to measure variations in thermal gradient across the walls of the cross-section and along the length of the superstructure resulting from different solar and environmental conditions. For the former purpose, single thermocouples were attached to the gauges at all gauge positions; for the latter, arrays of thermocouples were placed across the thickness of each wall element in two selected segments only. One of these was Segment P4-10E (River Torridge Bridge) in which the thermocouple arrays supplemented the vibrating wire gauges previously described. The other segment



(a)

(b)

Fig. 6 Positions of (a) thermocouples, and (b) electrical resistance strain gauges, in a segment of the Cogan Bridge.

selected for thermal investigation was in the Cogan Viaduct within the same half-span but additional to the three segments previously instrumented with vibrating wire gauges. This segment was also heavily instrumented with electrical resistance strain gauges placed in pairs near opposite faces of the walls for the purpose of monitoring short-term longitudinal and transverse strains due to diurnal temperature variations. Fig. 6 shows the positions of thermocouples and electrical resistance strain gauges in the Cogan Viaduct.

Lead wires of the installed gauges and thermocouples were bundled together and fixed securely with cable ties to the sides of reinforcement bars so as to avoid, as far as possible, contact with poker vibrators during concreting. The wire ends were protected during casting in two plastic trays suitably spaced off the main reinforcement of the inner faces of the webs so as to be flush with the inner surface of the finished concrete. The trays were later replaced by electrical switch boxes into which reading instruments could be plugged to obtain results.

3.3 Monitoring of pier reactions

After continuity has been established at mid-span the structure becomes indeterminate. Deformations due to subsequent time-dependent effects, settlement or superimposed loading will then result in the creation of additional secondary bending moments and reactions. In order to determine the magnitude of these effects, an attempt has been made to monitor pier reactions. To date this aspect of the investigation has been limited to Grangetown Viaduct. However, if it proves to be successful, the technique will also be applied during erection of Cogan Viaduct.

During construction of Grangetown Viaduct, each balanced cantilever was supported by a single 3m diameter column. After erection of the pierhead unit, the design required a balanced pair of segments to be permanently stressed prior to erection the next pair. Ten such pairs of segments were constructed per column. Finally, an additional segment was added leaving a gap of 300mm between adjacent cantilever tips to be filled by an in-situ joint. The bearings used for the two viaducts were designed with additional top and bottom plates, machined to match the bearings, to be cast into the soffit of the pierhead segment and the top of the column.

During installation of the two bearing plates at the top of the appropriate columns, a vibrating wire gauge was cast vertically immediately under the centre of each. The objective of introducting these two gauges was to monitor the strains due to the known self-weight of the individual segments during erection. It should then be possible to use these results as a calibration curve during subsequent loading and deformation of the structure. Three columns were instrumented in this way; the intermediate pier supporting the balanced cantilever containing the instrumented segments (Pier 3), together with the pier either side.

The results from the two gauges installed at Pier 3 of the Grangetown Viaduct are shown in Fig. 7. The shape of the two curves is similar, although the numerical values differ substantially from one another. The difference in the strain readings is thought to be due to the two gauges being located at at different positions relative to the bearings. The strain concentration immediately below the bearing plates change rapidly and even small variations in the position of the gauges will result in large variations in the recorded strain. At a later stage these results will be related to the corresponding dead load on the structure and will be utilised to monitor the pier reactions during the passage of a test load across the structure and during its early service life.

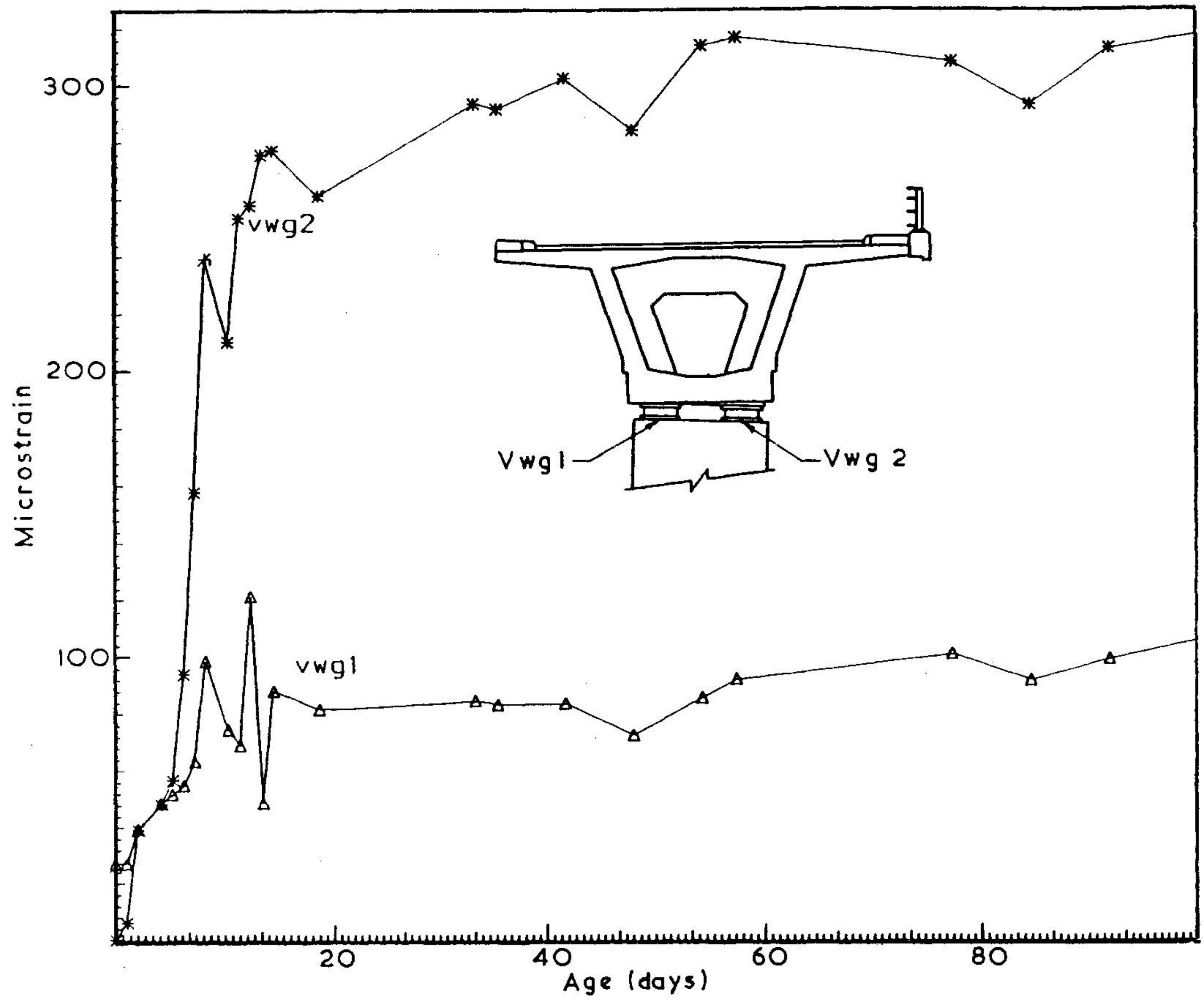


Fig. 7 Calibration curve for pier reactions

4. INITIAL RESULTS FROM STRAIN MONITORING

It was not possible to use remote automatic data logging equipment for a variety of reasons. These included difficult site conditions during erection, the distance between instrumented segments within the same span and the geographical location of the three structures. Approximately 20,000 strain readings from the bridge segments and prism tests were recorded manually in the first 12 months of the project. These have been entered in to a computer data base for subsequent reduction and analysis.

4.1 Shrinkage strains

Approximately 70% of the total expected shrinkage takes place in the first 28 days. The phenomenon is thus of much greater significance to insitu construction than to the glued segmental technique described here. In Fig. 8, two typical shrinkage strains recorded from Segment P4/10E in the River Torridge Bridge before erection are compared with those from two partially sealed prism specimens. As expected, one of the prisms stored outdoors displayed shrinkage strains approximately 15% higher than the other prism stored indoors at a relatively high humidity of 85%. Both prisms, however, over-estimated the true shrinkage strains as measured in the bridge segments by approximately 30%..

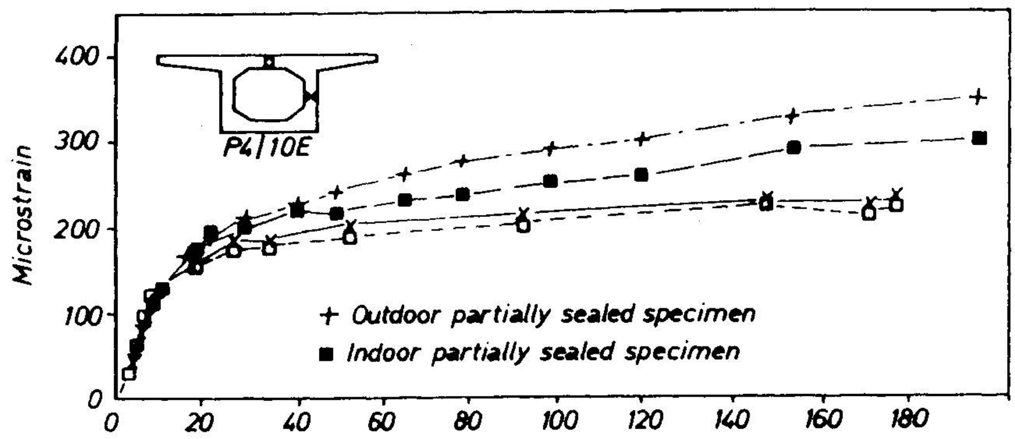


Fig. 8 Typical concrete shrinkage strains

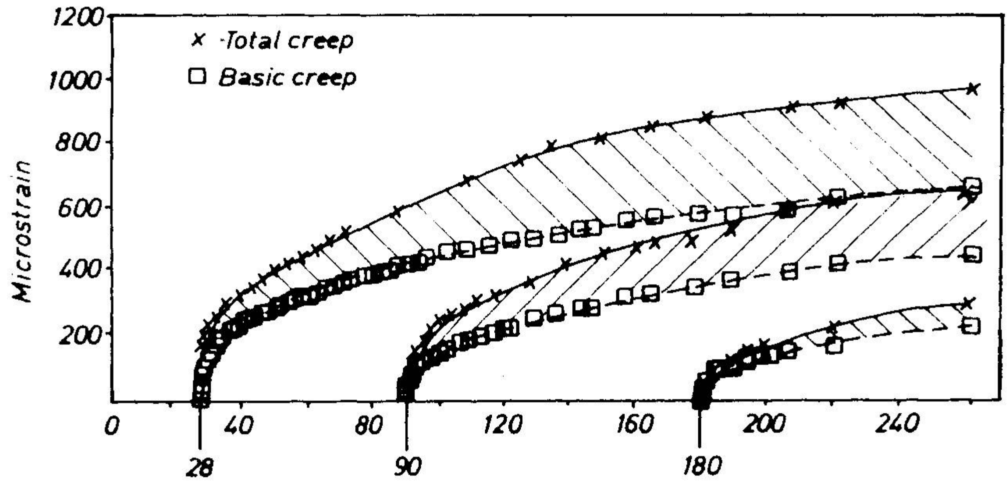


Fig. 9 Typical results from concrete creep tests

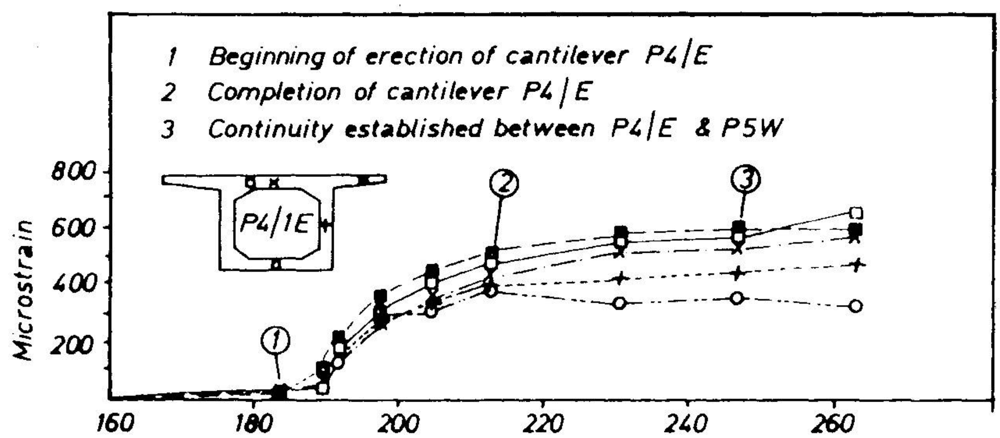


Fig. 10 Concrete strains measured during erection

4.2 Creep strains

Concrete creep is likely to be a more significant effect than shrinkage since it occurs as a result of self-weight stresses during erection and due to the subsequent application of external loads. Fig. 9 illustrates typical results of creep tests on prism specimens prepared with concrete from Segment P4/1E. From these results, corresponding to initial loading at ages of 28, 90 and 180 days, it is apparent that the level of creep strain is significantly reduced if the load is applied at a later age.

Two separate measures of creep are shown in Fig. 9. Basic creep is obtained from fully sealed specimens and may therefore be considered to occur in the absence of shrinkage. Total creep, which is approximately 30% higher, represents the creep of partially sealed specimens which thus enable the creep and shrinkage mechanisms to occur interactively.

4.3 Erection strains

Results from five typical gauges distributed around the cross-section of Segment P4/1E in the River Torridge Bridge are shown in Fig. 10 for the period including erection of Span 5. All strains are compressive and increase throughout erection of the balanced cantilever to a maximum of approximately 500 microstrain in the top flange. Thereafter, some change in these values of strain are apparent due to the combined effects of creep, shrinkage and the establishment of continuity with the previous span. The situation is further complicated by the application of additional construction loads caused by movements of the launching girder.

5. FORMWORK PRESSURE MEASUREMENTS

At the time of the manufacture of the 6m deep Cogan bridge pierhead segments, the authors and the engineer for the scheme were not aware of any work which would give an indication of the concrete pressures developed due to external vibration of the shutter. All the data available to date related only to static pressures developed on formwork during construction using internal vibrators. Hence a small additional study was initiated to monitor the formwork pressures during the manufacture of the deeper segments for the Cogan Viaduct.

5.1 Static pressure tests

In the first series of tests, all four load cells in the formwork were connected to a switch box, the concrete pressures being recorded on a digital readout unit. Four tests were carried out using the simple procedure illustrated in Fig. 11. The main conclusions were as follows:

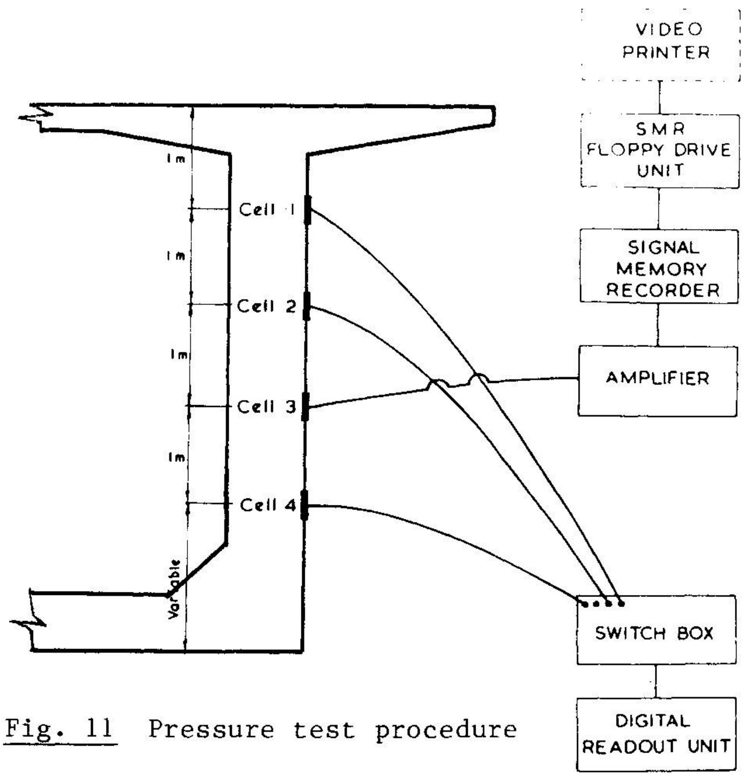


Fig. 11 Pressure test procedure

(a) The results were reproducible but did not show the dynamic pressures generated by external shutter vibration.

(b) Over short periods of time, the recorded mean concrete pressures were related directly to the hydrostatic head of concrete. Over longer periods, the pressures were reduced due to the setting of the concrete.

(c) The maximum pressures were recorded by Cell 3. The maximum pressure recorded was 72 kN/m^2.

5.2 Dynamic pressure tests

In a second series of pressure tests, an attempt was made to monitor the dynamic effects of the external vibration of the concrete. Since the maximum static pressures had been recorded by Cell No. 3, the instrumentation used in this part of the study was connected as shown in Fig. 11. In this case, Load Cell No. 3, was connected to a Signal Memory Recorder (SMR) via an amplifier. Although the SMR had the ability to monitor and register transient signals automatically, in this study it was triggered manually at the appropriate times when the shutter was being vibrated.

Typical results from these tests are given in Figs. 12 and 13. Fig. 12(a) shows a four-second burst of recorded pressure variations. The mean pressure recorded during this time was 17kN/m^2. By using a zoom facility it was possible to enlarge any part of the recording on the screen which allowed further analysis of small details to be carried out. Fig. 12(b) shows that the maximum pressure recorded was 23.0kN/m^2; the minimum pressure was 9.1kN/m^2.

Figs. 13(a) and (b) show the corresponding graphical displays for pressure variation when the mean pressure had increased to 74kN/m^2. Eight four-second bursts were recorded at various times during the manufacture of the elements; the pressures recorded are summarised in Table 2. The main conclusions from this part of the study are as follows:

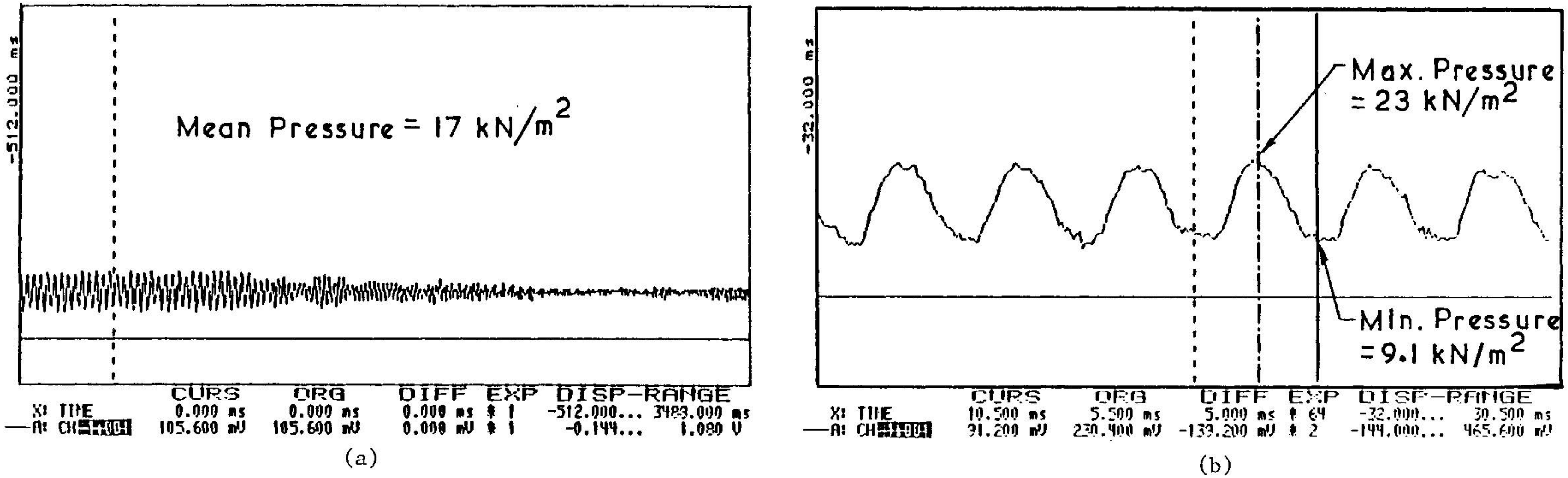
(a) Pressure variation does take place where external vibration is used.

(b) The range of pressure variations is at a maximum when the mean pressure is relatively low.

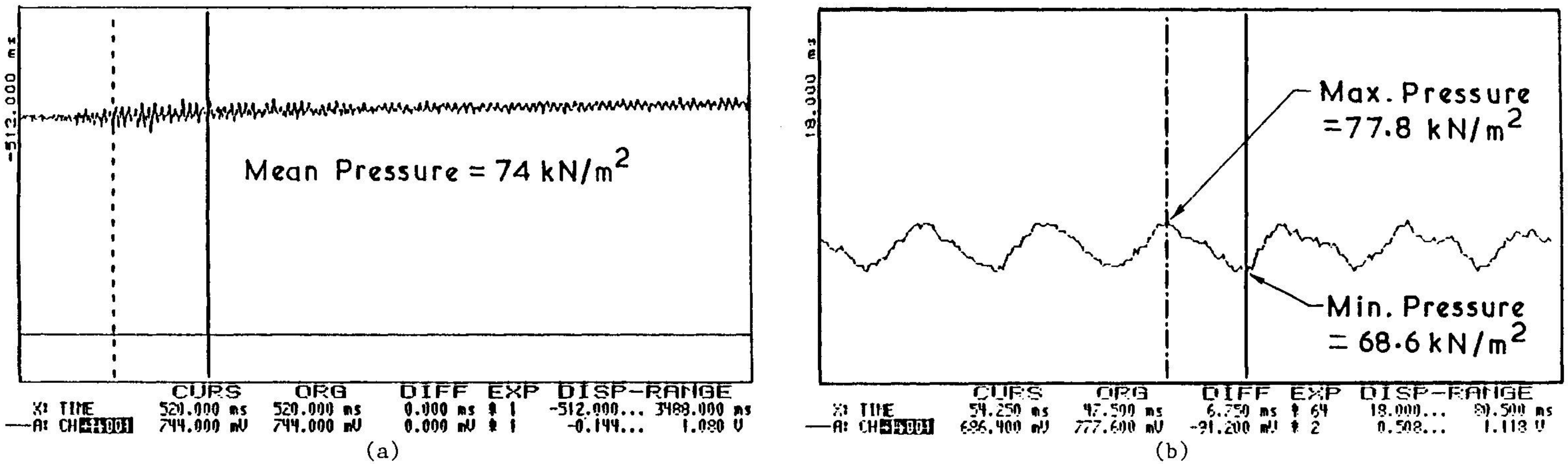
(c) The range of pressure variation is reduced as the mean pressure is increased. This could be due to a number of factors including the damping effect of the head of concrete and the setting of the concrete.

Test Number	Mean Pressure kN/m^2	Max. Pressure kN/m^2	Min. Pressure kN/m^2	Pressure Range kN/m^2
1	6.5	11.0	-1.9	12.9
2	17	23.0	9.1	13.9
3	57	63.4	51.4	12.0
4	74	77.8	68.6	9.2
5	75	78.7	73.9	4.8
6	75	76.8	73.4	3.4
7	74	74.9	73.0	1.9
8	74	74.9	72.0	2.9

Table 2 Pressure variations during external vibration



<u>Fig. 12</u> Results recorded at a mean pressure of 17kN/m^2



<u>Fig. 13</u> Results recorded at a mean pressure of 74kN/m^2

ACKNOWLEDGEMENTS

The field work described was funded by the Science and Engineering Research Council (UK) as part of its coordinated programme of large and full scale testing of structures. Additional support was provided by the County of South Glamorgan.

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